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Hydrodynamic and Structural Analysis of Flood Hazards at CPP-659 During a Peak Flow in the Big Lost River

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Hydrodynamic and Structural Analyses of Flood Hazards at CPP-659 During A Peak Flow in the Big Lost River

The following Engineering Design File (EDF) were prepared under the responsible charge of the Professional Engineer as indicated by the seal and signature provided on this page. The Professional Engineer is registered in the State of Idaho to practice Civil and Structural Engineering.

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1. Project File No.: ______________
2. Project/Task: CPP-659 Flood Hazard Analysis
3. Subtask: Hydrodynamic and Structural Analyses of Flood Hazards at CPP-659

4. Title: Hydrodynamic and Structural Analyses of Flood Hazards at CPP-659 During a Peak Flow in the Big Lost River

5. Summary: This summary briefly describes the problem to be addressed, gives a summary of the analyses performed in addressing the problem, and states the results, conclusions, and recommendations.

A study performed by the INEEL in 1986 estimated the flow volumes and water-surface elevations which occur during a peak flow in the Big Lost River at the INEEL. The INEEL study assumed that the 100-year peak flow and failure of Mackay Dam occur simultaneously, and estimated that the peak flow is equal to 28,500 ft^3/s at the diversion dam in the southwestern part of the INEEL. Building CPP-659—the New Waste Calcining Facility—lies within this hypothetical flood plain boundary based on the computed water elevation. The purpose of this analysis is to provide information to Idaho DEQ, in order to ensure compliance with RCRA regulations that require determination of hydrodynamic and hydrostatic forces expected to occur at the site and a description of flood protection devices at the facility and how these will prevent washout. The analysis consists of three parts: (1) A hydrodynamic analysis to compute the pressure exerted on the building by flood water; (2) A field investigation and structural analysis to determine whether the concrete foundation of CPP-659 can withstand the presence of flood water and to assess the likelihood of water infiltration; (3) A hydraulic analysis to examine the potential for sediment transport and erosion.

The results of this analysis lead to the following conclusions. Hydrostatic and hydrodynamic forces due to flood water above grade are negligible in comparison to lateral earth pressure. However, the weight of water in saturated soil considerably increases the lateral earth pressure. The lateral earth pressure of saturated soil was computed and shown to be 2 times larger than the pressure of dry soil. However, the strength of the below-grade retaining walls is adequate to support the increase in lateral earth pressure which may occur as a consequence of the flood postulated by the INEEL. Another major factor affecting the structural adequacy of the building is the method of construction, particularly the methods used to prevent water infiltration during a flood. A field investigation showed that construction of CPP-659 follows many of the methods described in the ACI Manual of Concrete Practice to assure a watertight structure. However, some minor water seepage was observed during the field investigation. Water accumulation is insignificant, which indicates that the rate of seepage is very low. Water that may seep into CPP-659 through pipe or utility penetrations is handled by flood protection devices that are designed to route water to the hot sump or valve cubicle so that water does not come into direct contact with waste piles or containerized hazardous wastes stored in the building. In particular, the flood protection devices are designed to preclude washout of hazardous waste from the building. Furthermore, a hydraulic analysis indicates that sediment transport and erosion at CPP-659 may occur. However, the likelihood of erosion is reduced by flood control devices that divert water to storage basins, asphalt and concrete that cover the gravel sediment found in the stream bed, and structures such as roads and buildings that slow and divert the flow.

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Introduction

In 1986, the INEEL published a report containing calculated flow volumes and water-surface elevations which occur during a peak flow in the Big Lost River at the INEEL (1). The INEEL study included the assumption that the 100-year peak flow and failure of Mackay Dam occur simultaneously, and thereby estimated that the peak flow in the Big Lost River is equal to 28,500 ft^3/s at the INEEL diversion dam. However, there are conflicting scientific opinions regarding the magnitude of the 100-year peak flow in the Big Lost River, and the INEEL Natural Phenomena Hazards Committee is currently addressing this issue. Presently, the water surface profile associated with a 28,500 ft^3/s flow is considered to be an upper bound on potential flooding at the INEEL. The particular water surface profile obtained from the INEEL study is used as a basis for the present analysis.

In the INEEL study, 57,740 ft^3/s was estimated to occur at Mackay Dam. The flow is attenuated downstream, and the INEEL diversion dam located in the southwestern part of the INEEL was estimated to receive 28,500 ft^3/s. The diversion dam was assumed to be unable to retain that flow, and so a large part of the discharge flows onto the site. The remaining water was assumed to flow through the diversion channel and into spreading areas. A hydraulic model was used to compute the flow volumes and water elevations within a 18 mile reach downstream of the diversion dam. Building CPP-659—the New Waste Calcining Facility at INTEC—lies within the hypothetical flood plain boundary that is based on computed water elevations given in the 1986 INEEL report (1).

The purpose of this engineering analysis is to provide information to Idaho DEQ regarding the hydrodynamic and structural effects of a peak flow. This analysis is performed to ensure compliance with RCRA regulations (2) that require an "engineering analysis to indicate the various hydrodynamic and hydrostatic forces expected to result at the site as a consequence of a 100-year flood," and "structural or other engineering studies showing the design of operational units and flood protection devices at the facility and how these will prevent washout." In the RCRA regulations (2), the term "washout" is defined as "the movement of hazardous waste from the active portion of a facility as a result of flooding."

This analysis is performed to ensure compliance with the following specific requirements stemming from application for a RCRA permit for mixed hazardous waste treatment in CPP-659 and to address issues presented in the DEQ letter received 9/27/00 requesting this study:

1. A description of building CPP-659 construction parameters which prevent run-on to the units described in the Volume 18 Part B permit application;
2. A professional engineer (PE) certification that CPP-659 could withstand hydrodynamic or hydrostatic forces applied to the building as a result of the hypothetical 100-year flood event described in the 1986 INEEL report (1);
3. PE certification that the design of operational units and/or flood protection devices in CPP-659 are adequate to prevent washout;
4. A discussion of the controls within the building that provide protection against washout.

This analysis consists of three parts:

1. Hydrodynamic and hydrostatic analyses were used to compute the pressure exerted on the building by stationary and moving flood water;
2. A field investigation and structural analysis are used to determine whether the concrete foundation of CPP-659 can withstand the presence of flood water and to assess the likelihood of water infiltration;
3. A hydraulic analysis is used to examine the potential for erosion and washout of hazardous waste.

Background

Peak Flow Analysis

Koslow and Van Haaften (1) examined the consequences of a failure of Mackay Dam and performed a hydraulic analysis to determine the extent of the flood plain for several scenarios. Their analysis included a predicted 100-year flood and simultaneous piping failure at Mackay Dam, which leads to a breach of the dam, overtopping of the INEEL diversion dam, and flooding of the INEEL site. This scenario results in a peak flow released from the dam that was calculated to be 57,740 ft^3/s. This flow between Mackay Dam and the INEEL is attenuated by storage, agricultural diversion, and channel infiltration. The calculated flow at the INEEL diversion dam is 28,500 ft^3/s. Since the diversion dam is unable to retain the high flow, most of the flood water is assumed to flow onto the site.

Flow Routing Analysis

The peak flow estimated by Koslow and Van Haaften (1) was used in a flow routing analysis to determine the extent of the flood plain at the INEEL site. The geometry of the channel was determined from USGS topographical maps, and the Big Lost River stream bed was examined to determine surface roughness. The Bernoulli equation for ideal flow and the Manning relation for energy loss in open channels were used to compute the peak flow and water elevation at each cross-section. The INTEC site was surveyed by INEEL engineers to determine building and ground elevations. All vertical elevations are in reference to the National Geodetic Vertical Datum of 1929 (NGVD29). Of particular interest in this study is Building CPP-659 located at the INTEC facility. The leading edge of the flood wave is estimated to arrive at INTEC approximately 17.1 hours after breach of the dam. The peak flow is attenuated to 24,870 ft^3/s, and the peak water velocity is estimated to be 2.2 ft/s. Since the area surrounding INTEC is very flat, flood water will spread easily and so the flood plain is wide and shallow. The elevation of the stream bed is 4911 feet and the calculated water elevation is 4916 feet. The lowest ground elevation at CPP-659 is 4912.1 feet and occurs at the east side of the building. These results suggest that the depth of flood water may reach 4 feet at the building's foundation. Therefore, a water depth equal to 4 feet is used in the following hydrodynamic and hydrostatic analyses.

Koslow and Van Haaften (1) also performed an analysis to examine the potential for overland flooding due to localized heavy rain and snowmelt. It was found that localized flooding due to a 25-year peak rainfall and simultaneous snowmelt lead to a peak flow equal to 32 ft^3/s. This runoff can be accommodated by the drainage basin at INTEC and flood control devices such as culverts, dikes, and ditches. Meanwhile, flood water may collect in low-elevation areas at

INTEC. The following hydrodynamic and hydrostatic analyses of 4 feet of water at the foundation of CPP-659 may also be used to assess the effect of overland flooding due to localized precipitation.

Hydrostatic and Hydrodynamic Analyses

Hydrostatic Forces

The results of the INEEL study (1) were used to determine that the depth of flood water may reach 48 inches at the CPP-659 building foundation during a peak flow in the Big Lost River streambed adjacent to INTEC. At a depth of 48 inches, the hydrostatic pressure on the foundation is

$$P_{water} = \gamma_{water} \cdot d = 62.4 \frac{lb}{ft^3} \cdot 4\ ft = 249.6 \frac{lb}{ft^2},$$

where P_{water} is the hydrostatic pressure, γ_{water} is the weight of water, and d is the water depth. The resultant force per unit width of foundation is

$$F_{water} = \frac{1}{2} P_{water} \cdot d = 499.2 \frac{lb}{ft},$$

where F_{water} is the resultant force that occurs at a height above grade equal to $d/3$, as is shown in Fig. 1.

The lateral earth pressure of saturated soil includes the effect of water pressure and soil pressure. The at-rest earth pressure due to the weight of soil is

$$P_{soil} = K_o (\gamma_{sat} \cdot H - \gamma_{water} \cdot H) = 0.375 \cdot \left(135 \frac{lb}{ft^3} - 62.4 \frac{lb}{ft^3}\right) \cdot H = 27.2 \frac{lb}{ft^3} \cdot H,$$

where P_{soil} is the earth pressure, γ_{sat} is the weight of saturated soil, H is the soil depth, and K_o is the earth pressure coefficient. The at-rest earth pressure coefficient was obtained from the relation

$$K_o = 1 - \sin\phi,$$

where ϕ is the angle of internal friction which is equal to 43° according to the NWCF soils report (3). The weight of saturated soil at NWCF is assumed to be equal to the weight of dense, mixed-grain sand given by Peck et al (4). The resultant force per unit width is

$$F_{soil} = \frac{1}{2} P_{soil} \cdot H = 13.6 \frac{lb}{ft^3} \cdot H^2,$$

where F_{soil} is the resultant force that occurs at a height equal to $H/3$ from the base of the retaining wall. The hydrostatic pressure due to the presence of water is

$$P_{\text{wet soil}} = \gamma_{\text{water}} \cdot H = 62.4 \frac{\text{lb}}{\text{ft}^3} \cdot H,$$

where $P_{\text{wet soil}}$ is the hydrostatic pressure. The resultant force per unit width of retaining wall is

$$F_{\text{wet soil}} = \frac{1}{2} P_{\text{wet soil}} \cdot H = 31.2 \frac{\text{lb}}{\text{ft}^3} \cdot H^2,$$

where $F_{\text{wet soil}}$ is the resultant force that occurs at a height equal to $H/3$ from the base of the retaining wall. The total resultant force per unit width of retaining wall is

$$F_{\text{total}} = F_{\text{soil}} + F_{\text{wet soil}} = 44.8 \frac{\text{lb}}{\text{ft}^3} \cdot H^2,$$

where F_{total} is the total resultant force that occurs at a height equal to $H/3$ from the base of the retaining wall, as is shown in Fig. 1.

In the case of dry soil, the resultant force per unit width of retaining wall is

$$F_{\text{dry soil}} = \frac{1}{2} \cdot K_o \cdot \gamma_{\text{dry}} \cdot H^2 = \frac{1}{2} \cdot 0.375 \cdot 118 \frac{\text{lb}}{\text{ft}^3} \cdot H^2 = 22.1 \frac{\text{lb}}{\text{ft}^3} \cdot H^2.$$

The density of dry soil is given in the NWCF soils report (3).

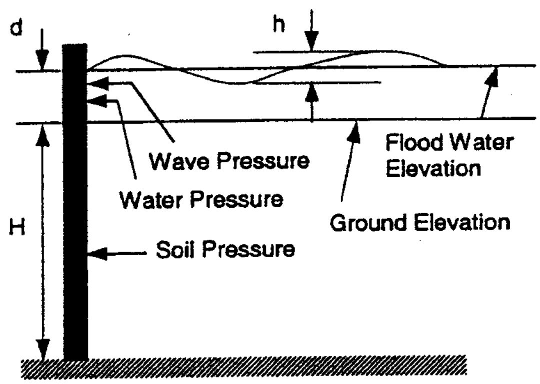

Fig. 1. Various forces acting on a retaining wall during a flood.

Hydrodynamic Forces

The force of moving flood water is calculated by considering the impact of shallow water waves caused by a high wind. A graph that shows the relation between wind velocity, water depth, wave height, and wave period is given in Fig. 10-16 on page 10-36 in Brater and King (5). Assuming a wind velocity equal to 60 mph and a water depth equal to 4 feet, the graph shows that the wave height is 2.0 feet and the wave period is 3.4 seconds. The relation between wave period and wavelength of shallow water waves is

$$\frac{L}{T} = \sqrt{g \cdot d},$$

where L is the wavelength, T is the wave period, d is the water depth, and g is the gravitational acceleration. Assuming a water depth equal to 4 feet and a wave period equal to 3.4 seconds, the wavelength is

$$L = T\sqrt{g \cdot d} = 3.4\,\mathrm{s}\sqrt{32.2\frac{\mathrm{ft}}{\mathrm{s}^2} \cdot 4\,\mathrm{ft}} = 38.6\,\mathrm{ft},$$

and the wave velocity is

$$\frac{L}{T} = \frac{38.6\,\mathrm{ft}}{3.4\,\mathrm{s}} = 11.35\frac{\mathrm{ft}}{\mathrm{s}}.$$

In comparison, the velocity of flood water as estimated by Koslow and Van Haaften (1) is 2.2 ft/sec. Therefore, the velocity of moving flood water is small in comparison to the velocity of wind-generated waves.

The resultant force per unit width of retaining wall, which is caused by wind-generated waves, is calculated from an empirical relation described on page 10-41 in Brater and King (5). Assuming a wave height equal to 2.0 feet, the pressure exerted by the wave is

$$P_{wave} = \gamma_{water} \cdot h = 62.4\frac{\mathrm{lb}}{\mathrm{ft}^3} \cdot 2.0\,\mathrm{ft} = 124.8\frac{\mathrm{lb}}{\mathrm{ft}^2}.$$

According to Fig. 10-21 on page 10-42 in Brater and King (5), the pressure distribution is uniform from the ground to the still-water height, and hydrostatic from the still-water height to a height above still water equal to $1.66 \cdot h$. Assuming a water depth equal to 4 feet, the force of the wave is

$$F_{wave} = P_{wave}\left(d + 0.5 \cdot 1.66 \cdot h\right) = 124.8\frac{\mathrm{lb}}{\mathrm{ft}^2}\left(4\,\mathrm{ft} + 0.5 \cdot 1.66 \cdot 2.0\,\mathrm{ft}\right) = 706.4\frac{\mathrm{lb}}{\mathrm{ft}},$$

and occurs at a height above grade equal to 2.9 feet, as is shown in Fig. 1.

The results of these calculations show that the hydrostatic and hydrodynamic forces are small in comparison to the lateral earth pressure. Furthermore, hydrostatic and hydrodynamic forces have a negligible effect on the overturning moment. However, a substantial increase in the earth pressure occurs when the soil becomes saturated—the dry soil force is equal to $22.1 \cdot H^2$ lb/ft and the saturated soil force is equal to $44.8 \cdot H^2$ lb/ft. Since the topmost 40 feet of soil at the NWCF is mostly sandy gravel that is dry and permeable (3), the assumption of saturated soil may be very conservative. Therefore, the calculated earth pressure is an upper bound on the actual earth pressure that would occur during a flood.

Structural Analysis

The second and third levels of CPP-659 are below grade and contain an inner cell structure encased in a 4 feet thick shielded concrete wall. The cell structure is surrounded by corridors and various utility rooms. The retaining walls on the second and third levels support gravel backfill and are 1 1/3 to 2 feet thick concrete. The first level is 3 feet above grade and contains a maintenance area leading to the cell structure and an outer office area built on gravel backfill. The exterior retaining wall supporting the first level is 1 1/6 feet thick concrete.

The structural features of the concrete foundation at CPP-659 were examined during a field investigation. The following features were examined: footing and foundation structures; type of concrete used during construction; soil grading and drainage systems; exterior wall construction, including joints and the method of sealing penetrations; openings such as doorways that enable water to easily infiltrate; the use of water stops and sealant to prevent water infiltration; and the occurrence of water seeping through cracks and penetrations. The ACI Manual of Standard Practices (6) provides guidance on construction of watertight concrete structures. The result of the field investigation shows that construction of the NWCF follows many of these standard practices, though some minor water seepage was observed.

The following list of construction practices were used to assure a watertight foundation and to provide adequate drainage during a flood.

(1) The retaining walls that support lateral earth pressure were made using high-density, low-permeability concrete.

(2) Soil surrounding the foundation is graded to slope away from the building.

(3) All joints are fitted with carbon steel water stops to prevent water infiltration.

(4) The first level is at an elevation higher than the flood water elevation.

(5) Visible cracks in the above-grade, exterior concrete foundation were not observed.

(6) Water entering the building drains to the hot sump tank located below the third level.

The following list of observations suggest the potential for water infiltration during a flood, particularly seepage caused by water infiltration through pipe penetrations and other openings. The field study investigated the potential for water infiltration through the utility piping tunnel, tank farm waste pipe, concrete hatches, doorways and other openings.

(1) All the INTEC utility piping is carried in an underground tunnel that sometimes contains water because the tunnel has manholes that provide an opening for runoff. Despite the presence of level alarms and a pump in the utility tunnel, the water in the utility tunnel occasionally seeps into the utility corridor located at the second level of CPP-659. Seepage occurs through pipe penetrations into the utility corridor. Seeping water is collected by a floor drain in the utility corridor and flows into VES-NCC-122—the non-fluoride hot sump tank. This tank has a maximum capacity of 4300 gallons and is equipped with level monitoring and control equipment. If seeping water enters the building as a result of a prolonged flood event, VES-NCC-122 can be sampled and its contents transferred to VES-

WL-133—the Process Equipment Waste Evaporator (PEWE) feed tank. The contents of tank VES-NCC-122 can also be transferred to VES-NCC-119—the fluoride hot sump tank—or the Tank Farm Facility if the PEWE cannot accept the liquid. Water is transferred through steam jets that have a capacity equal to 20 gal/min each.

(2) A 3 inch stainless steel pipe carrying waste from the tank farm is the only pipe that penetrates the inner cell structure. This pipe is encased in a larger pipe that is well sealed, and water infiltration through the pipe penetration has not been observed. If flood water enters the building at this location, the flood protection devices are designed to route water to a sump in the valve cubicle which is equipped with leak detection devices. From this sump the water may be pumped to a variety of other tanks, such as VES-NCC-119, VES-NCD-123, and VES-NCD-129. These tanks are equipped with level monitors and overflow alarms.

(3) Concrete hatches located in the maintenance area at the first level lead to the cell structure. These hatches are not watertight, but flood water will not reach the maintenance area since the first level is at an elevation equal to 4917 feet, which is 1 foot higher than the flood level.

(4) The lowest elevation of an entry into the building is a doorway on the north side, which is at an elevation equal to 4914.3 feet. This is 1.7 feet less than the flood water elevation. The doorway leads to the first level, which is at an elevation equal to 4917 feet. The exterior retaining wall at this doorway is located 20 feet from the closest retaining wall on the second and third levels. If a person enters the exterior door at the north side of CPP-659, he must walk up steps to an elevation of 4917 feet to reach the first level, and then walk 20 feet toward the center of the building to be above the second level. Therefore, water entering the exterior doorway may only infiltrate the gravel backfill underneath because there is no path for water to infiltrate the levels below grade.

Another important consideration is the ability of the retaining walls to withstand lateral earth pressure. In the section on hydrodynamic analysis, the at-rest lateral earth pressure of saturated soil was computed and shown to be 2 times larger than the pressure of dry soil. This particular flood hazard affects all below-grade retaining walls that support backfill. The structural design of the second and third levels of CPP-659 is complex, and the concrete retaining walls have a variable height, width, and thickness. Surcharge loads are present in addition to lateral earth pressure. Furthermore, the strength of reinforced concrete depends on the exact size, number, and placement of the steel bars. Therefore, a thorough assessment of the effect of soil saturation on the stress in retaining walls is a complex structural analysis that is beyond the scope of this study. However, the following simple calculation demonstrates that the strength of the below-grade retaining walls are more than adequate to support the increase in lateral earth pressure which may occur as a consequence of a flood.

The building structure consists of two levels below grade, and the height of each level is 17 feet. The first level is 3 feet above grade. Therefore, the depth of soil at the base of the first level retaining wall is 14 feet. Consider a concrete beam fixed at both ends and acted on by a distributed force, as is shown in Fig. 2. This particular beam loading represents the lateral earth pressure acting on a section of retaining wall, and leads to a conservative estimate of the shear force and the bending moment. To examine the loading on the weakest section of retaining wall, assume that the length of the beam is equal to 8 feet—the maximum spacing between supports—

and the thickness of the beam is equal to 16 inches—the minimum thickness of the foundation walls. Using the results calculated previously in the section entitled *Hydrostatic Forces*, the force per unit area of beam is equal to

$$P = P_{soil} + P_{wet\ soil} = 27.2\frac{lb}{ft^3} \cdot H + 62.4\frac{lb}{ft^3} \cdot H = 89.6 \cdot H\, lb/ft^2,$$

where H is measured in feet. At the base of the beam where H is equal to 14 feet, the pressure is equal to $1250\, lb/ft^2$. To examine the maximum loading on the beam, assume that this pressure is uniformly distributed on the entire length of the beam.

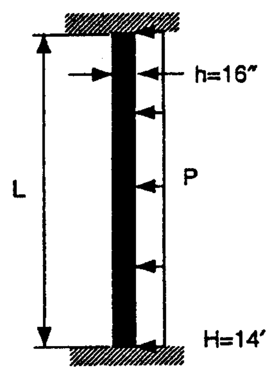

Fig. 2. Lateral earth pressure acting on a retaining wall.

The maximum shear force and bending moment occurs at the ends of the beam, and are obtained from the following formulas found in ACI 318 (7):

$$M = \frac{P\,L^2}{12} = 6690\frac{ft - lbs}{ft},$$

$$V = \frac{P\,L}{2} = 5020\frac{lbs}{ft}.$$

The actual force and moment are multiplied by a load factor equal to 1.7, as specified in ACI 318 (7), to give $M_u = 11{,}400\, lb\, ft$ and $V_u = 8{,}500\, lb$ per 1 foot width of beam.

To compute the allowable shear and moment capacity of the concrete beam, assume the minimum required reinforcement according to ACI 318-77 (7), which was the building code for reinforced concrete at the time the NWCF was built. For reinforcement with a yield strength equal to 40,000 psi, assume #4 bar spaced 8 inches center to center, and assume top and bottom covers equal to 1 inch. This meets the requirements that the area of vertical reinforcement shall not be less than 0.0015 times the wall area and the reinforcement layers shall not be placed more than 1/3 the wall thickness from the surface, as described in Sections 14.2.11 and 14.2.12 of ACI 318 (7). Furthermore, the concrete is assumed to have a compressive strength equal to 4000 psi.

The computation of moment and shear capacity are based on ACI 318 (7) and the CRSI Design Handbook (8). The shear capacity is obtained from Section 11.3.1.1 of ACI 318 (7):

$$V_c = 0.85\, 2\sqrt{f_c'}\, b\, d\,,$$

where f_c' is the compressive strength of concrete, b is the width of the beam, and d is the distance from the extreme compression fiber to the centroid of the tension reinforcement. The moment capacity for a single layer of tension reinforcement is obtained from page 5-7 in the CRSI Design Handbook (8):

$$M_n = 0.90\, A_s\, f_y\, (d - a/2)\,,$$

where A_s is the area of tension reinforcement, f_y is the yield strength of the reinforcement, and a is the depth of the concrete compression block which is obtained from a balance of concrete compression and bar tension:

$$A_s\, f_y = 0.85\, f_c'\, b\, a\,.$$

The moment capacity and shear capacity for the concrete beam are $M_n = 13{,}100$ lb ft and $V_c = 19{,}000$ lb per 1 foot width of beam, which are larger than the factored moment and shear computed above. In fact, the retaining walls at the NWCF are stronger than this simple example indicates, owing to the presence of intersecting walls, columns, and slabs anchored to each section of retaining wall.

Hydraulic Analysis

Transport of sediment caused by moving flood water may lead to erosion of the stream bed. The type of soil needs to be known to assess the potential for erosion. A previous study (3) found that the topmost 40 feet of soil at the NWCF is mostly sandy gravel and some silt. Below the topmost layer is a 0 to 10 feet intermediate layer of clay soil containing silt and sand, and below the intermediate layer is basalt bedrock. A sieve analysis performed on the topmost layer of soil showed that the 75th percentile of the particle diameter distribution is approximately equal to 0.4 inches to 0.8 inches (3), which means that 75% of the particles by weight are that size or finer. In the case of a non-cohesive soil with a particle diameter larger than approximately 0.05 inches, the critical shear stress for sediment transport may be obtained from the following relation given on page 7-26 in Brater and King (5):

$$\tau_{critical} = 0.4 \cdot D\,,$$

where D is the 75th percentile of the particle diameter distribution measured in inches, and the critical shear stress is measured in lb/ft^2. Assuming a particle diameter equal to 0.6 inches,

$$\tau_{critical} = 0.4 \cdot 0.6\ \text{in} = 0.24 \frac{\text{lb}}{\text{ft}^2}\,.$$

The shear stress due to uniform flow of water in a channel having a small slope may be obtained from the following relation given on page 7-25 in Brater and King (5):

$$\tau = \gamma_{water} \cdot d \cdot s,$$

where d is the water depth and s is the channel slope. In the Big Lost River stream bed near INTEC, the channel slope is approximately equal to 16 feet per mile. Since the flood water depth at CPP-659 is approximately equal to 4 feet, the shear stress due to moving flood water near the building is

$$\tau = 62.4 \frac{\text{lb}}{\text{ft}^3} \cdot 4\text{ ft} \cdot \frac{16}{5280} = 0.76 \frac{\text{lb}}{\text{ft}^2}.$$

Since $\tau > \tau_{critical}$, compute the particle size needed to resist erosion. Assuming a particle diameter equal to 2.0 inches,

$$\tau_{critical} = 0.4 \cdot 2.0\text{ in} = 0.80 \frac{\text{lb}}{\text{ft}^2}.$$

Therefore, erosion of sand and rock with a diameter smaller than 2 inches may occur. However, the likelihood of erosion is greatly reduced because much of the sandy gravel sediment found in the stream bed has been covered with asphalt and concrete at INTEC. Furthermore, the likelihood of erosion is reduced by the presence of flood control devices that divert water to storage basins and structures such as roads and buildings that slow and divert the flow. Since the main foundations are deep and the gravel has some larger rock, erosion of the soil is not likely to cause damage to critical structural components.

Conclusions

An engineering analysis was used to calculate the various hydrodynamic and hydrostatic forces expected to result at Building CPP-659 as a consequence of a 100-year flood coinciding with a failure of Mackay Dam. A structural study was used to describe the design of CPP-659 and its flood protection devices and how these will prevent washout of hazardous waste. Specific details are given below.

An engineering analysis was used to determine whether CPP-659 can withstand a peak flow in the Big Lost River adjacent to INTEC. Hydrostatic and hydrodynamic forces due to flood water above grade are negligible in comparison to lateral earth pressure, but the weight of water in saturated soil considerably increases the lateral earth pressure. In fact, the lateral earth pressure of saturated soil was computed and shown to be 2 times larger than the pressure of dry soil. However, the strength of the below-grade retaining walls is adequate to support the increase in lateral earth pressure which may occur as a consequence of a flood.

Another major factor affecting the structural adequacy of the building is the method of construction, particularly the methods used to prevent water infiltration during a flood. A field investigation showed that construction of the NWCF follows many of the methods described in the ACI Standard Practices to assure a watertight structure. Furthermore, the field investigation examined the potential for water infiltration through the utility piping tunnel, tank farm waste pipe, concrete hatches, doorways and other openings. The first level of CPP-659 contains a pathway for water to enter the concrete hatches in the maintenance area. Since the elevation of the first level is one foot above the elevation of the hypothetical 100-year flood, water entry through these openings will not occur. However, some minor water seepage from a below-grade utility tunnel was observed during the field investigation. The rate of water seepage is very low and water accumulation is insignificant. Water that may seep into CPP-659 through pipe or utility penetrations is handled by flood protection devices that are designed to route water to the hot sump tank so that water does not come into direct contact with waste piles or containerized hazardous wastes. In particular, the flood protection devices are designed to preclude "washout" or movement of hazardous waste from the building as a result of flooding.

Another issue concerns the potential for erosion and sediment transport. The shear stress of moving flood water near CPP-659 is larger than the critical shear stress needed to cause sediment transport, and so erosion at CPP-659 may occur. However, the likelihood of erosion is greatly reduced because much of the sandy gravel sediment found in the stream bed has been covered with asphalt and concrete at INTEC. Furthermore, the likelihood of erosion is reduced by the presence of flood control devices that divert water to storage basins and structures such as roads and buildings that slow and divert the flow. Since the main foundations are deep and the gravel has some larger rock, erosion of the soil is not likely to cause damage to critical structural components.

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